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THE PROBLEMS OF CIVIL AND STRUCTURAL DESIGN OF STEAM ELECTRIC PLANTS FOR THE PACIFIC GAS AND ELECTRIC COMPANY IN CENTRAL CALIFORNIA

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CONSTRUCTION DIVISION

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THE PROBLEMS OF CIVIL & STRUCTURAL DESIGN OF STEAM ELECTRIC PLANTS FOR THE PACIFIC GAS AND ELECTRIC COMPANY IN CENTRAL CALIFORNIA AT ANTIOCH & AT PITTSBURG

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Engineers & Constructors

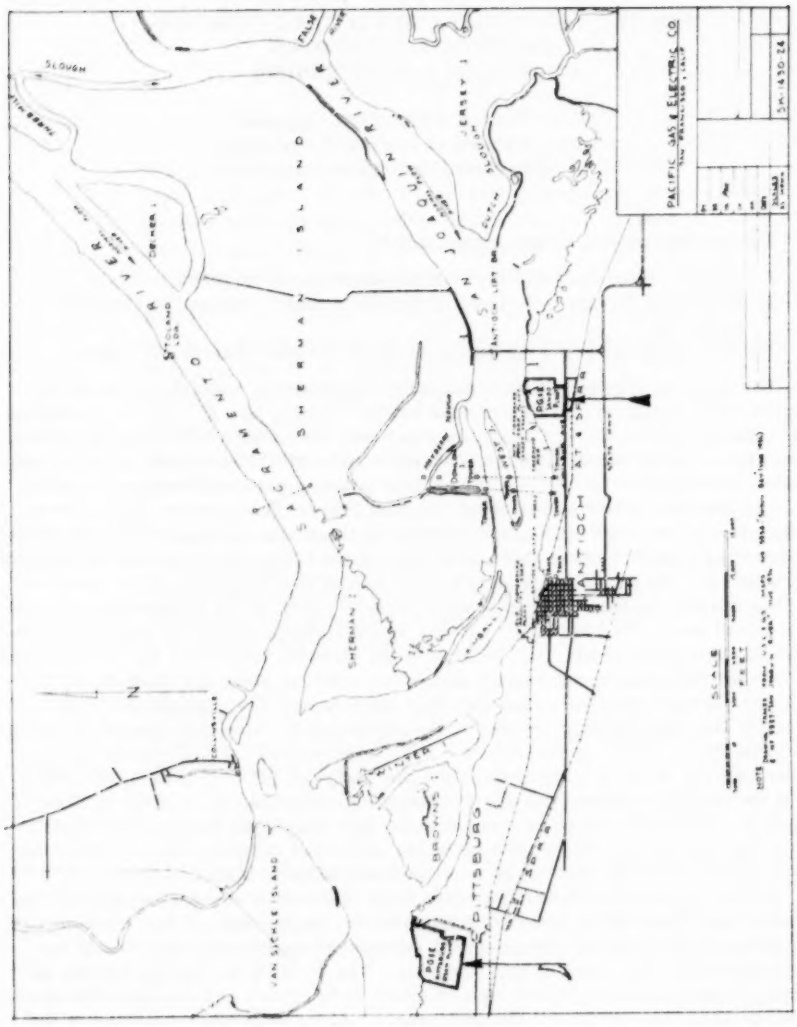
The general outline of this paper will be:

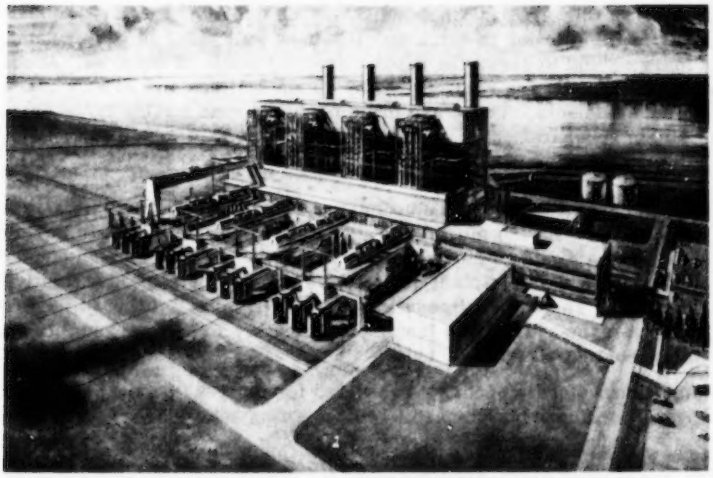
1. General electrical and mechanical features of the plant.
2. Outline of the various items of design of major interest to Civil Engineering.
3. Discussion as separate items, of those various chapters or topics.

This portion of the expansion construction program consists of an initial power plant designed and constructed for the Pacific Gas & Electric Company at Antioch, Contra Costa County, to which has been added additional facilities, and also a sister or big twin plant which is to be at Pittsburg and which is now under construction to augment the earlier power plant construction program.

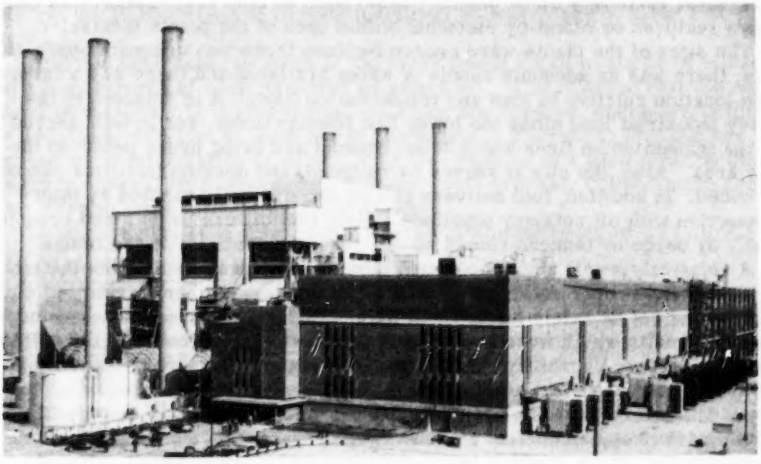
The steam electric power plant construction program in the West is very important to us since such plants represent the major portion of the new power generating capacity being made available by the tremendous expansion program in which the Pacific Gas & Electric Company is now engaged. As a result of this program, by the end of 1952 approximately 60% of the tremendous system peak load was to be carried by steam. In 1940 steam plants carried less than 25% of the system peak. The portion of the kilowatt hours that will be supplied by steam will, of course, depend upon the amount of snow and rain in any particular year, but in 1949 more than half the kilowatt hours produced by the Pacific Gas and Electric Company was developed in its steam plants, whereas in 1940 steam plants generated only 10% of the energy. From this it can be seen that the P. G. & E. system is not one supplied primarily by hydro-electric generating stations, but steam plants are required, not merely as back-up to be used in the event of transmission line failure or for dry-year operation, but also as the major portion of the base load capacity; and as such, the steam units must be capable of sustained operation at high efficiency.

A brief description should be made here of the size of the program and the general area served in order to emphasize the importance of this construction program, both from the standpoint of increased capacity and because of the influence upon the speed of this program. The P. G. & E. System serves an area of approximately 90,000 square miles in Northern and Central California with a population of about 5,000,000 people. Along with other sections of the West Coast, this area, as is well known, is experiencing an unusual growth in population, in industry, and in the general level of business and agricultural activity which reflects in a rapidly increasing demand for electric power. Since the early thirties the peak load on the P. G. & E. System has been climbing steadily. During the war years the growth of the need was accelerated and


PACIFIC GAS & ELECTRIC CO
SAN FRANCISCO, CALIF.
SAN JOAQUIN RIVER
SACRAMENTO RIVER
SHERMAN ISLAND
ANTIOCH
PITTSBURG
SHERMAN I.
BROWNS I.
JAN SICKLE ISLAND
SLOUGH
SCALE
FEET



STEAM ELECTRIC GENERATING PLANT FOR PACIFIC GAS & ELECTRIC COMPANY AT PITTSBURG, CALIFORNIA. ARTIST'S PRESENTATION OF THE PLANT, CONSTRUCTION OF WHICH WAS BEGUN LATE IN 1951 BY FIELD CREWS OF BECHTEL CORPORATION. THE PLANT AS SHOWN CONSISTS OF 4 "OUTDOOR" 150,000 KW TURBO-GENERATOR UNITS, AN ENCLOSED CONTROL BAY, A ROOFED PORTION OVER THE BOILERS, AND THE OFFICE, MACHINE SHOP AND WAREHOUSE FACILITY.



STEAM ELECTRIC GENERATING PLANT FOR PACIFIC GAS & ELECTRIC COMPANY AT ANTIOCH, CONTRA COSTA COUNTY, CALIFORNIA. THE FIRST STAGE OF CONSTRUCTION HOUSING THE THREE 110,000 KW UNITS IS SHOWN PRACTICALLY COMPLETED, WITH THE SECOND STAGE AT THE RIGHT, UNDER EARLY CONSTRUCTION; THE STEEL FRAME ERECTED AND THE MASONRY WORK JUST STARTING.

after the war years has continued to accelerate. The construction program, to keep up with this increase in demand, lagged during the war years and now must accelerate in order to catch up. It is now well along to restore adequate power margins and to serve the rapidly growing needs of the area. About 75% of the new capacities are represented by steam stations and 25% by hydro-electric developments. There are three basic reasons for the large proportion of steam generation:

1) Steam capacity additions were postponed by the war.

2) Steam capacity is required because of the changing economic steam hydro ratio. As the more favorable hydro-electric sites are developed, those remaining are more costly. At the same time, technological developments in steam plant design have tended to reduce the relative cost of steam. The over-all result has been a gradual shifting of the economic steam-hydro ratio from approximately 75% hydro and 25% steam to approximately a 50-50 ratio.

3) Additional steam capacity is required for the peak support of additional hydro developments planned by the company after 1952.

The new steam station at Antioch, Contra Costa, and others similarly planned at the same time, were designed for system as well as local area use, and it was necessary in each case to provide full capacity outlets directly to the high voltage system. Also, these plants were required to give full consideration to interchange and interconnection of elements within the plant for standby operation. Subsequent developments, such as the second addition at Antioch and the large plant at Pittsburg, were designed on the basis of system security and inter-connection; that is, it was not so important that any one particular plant be entirely self-sufficient because the system could tie together several relatively large plants. This required less inter-connection and safety features or stand-by elements within each of the plants themselves.

The sites of the plants were chosen because there was adequate space available, there was an adequate supply of water available and there was a favorable location relative to load and transmission lines. It is adjacent to the heavy industrial load along the lower San Joaquin River, and is well served by the transmission lines which inter-connect and bring hydro power to the Bay area. Also, the site is served by railroad, and docking facilities can be provided. In addition, fuel delivery can be satisfactorily handled by interconnection with oil refinery pipelines. Also, fuel oil can be obtained economically by barge or tankers should such a requirement arise in the future.

A relatively small amount of earth moving was required to make the site level and at a proper elevation to utilize low head condenser circulating water pumps, and yet be safely above flood waters. The first three main turbine generator units which were installed at Contra Costa are rated at 110,000 kilowatts maximum capability. In addition, there are three turbine generators rated at 7,500 kilowatts each for house service supply. It is interesting to compare the total capacity of the first three units which would constitute a single plant of approximately 330,000 kw with that of the Shasta plant of the Central Valley Project which has a capacity of 375,000 kilowatts. The presently authorized capacity of the Contra Costa plant, including the two additional generators now under construction is about 560,000 kilowatts maximum rating. The first stage of construction, that is, the first three units, cost about $50,000,000 and the second stage of construction at Contra Costa about $30,000,000 more. The Pittsburg plant will cost on the order of $80,000,000.

After listing the capacities and costs of some of these plants it would be interesting at this time to refer again to the comparative cost of hydro power and steam power. The relative costs for a kilowatt of capacity are: for steam at Contra Costa- $140; for hydro power on the Kings plant- $260. Fixed charges, such as interest, taxes, etc., are higher for hydro electric capacity, but of course no fuel is required. The steam plant on the other hand may consume nearly $8,000,000 worth of fuel in a year and, in addition, will require more operation and maintenance labor. When capital costs are adjusted for these elements, there is a balance of only about $20.00 a kilowatt in favor of the hydro development on the Kings plant. There are many factors to be taken into account in comparing steam with hydro power. In some instances steam will be more favorable, and in others hydro will have the lower annual capitalized cost. In the future, as in the past, both sources of energy will be developed in the P. G. & E. system.

Outline of the Items of Civil Design of the Plant at Antioch and at Pittsburg

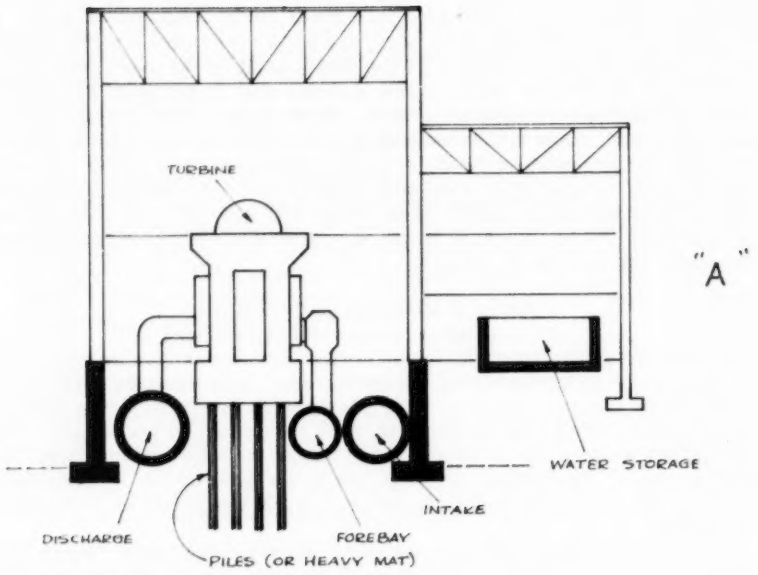
Many of the problems which are encountered in the structural design of power plants can be classed as "heavy industrial type." However, some of these normal, ordinary problems are made interesting by the unusual size of the facility involved, for example: Most industrial buildings normally contain bridge cranes. The interesting point to a power plant crane, as at Contra Costa, is the fact that it must lift and place a 200 ton weight very accurately in position.

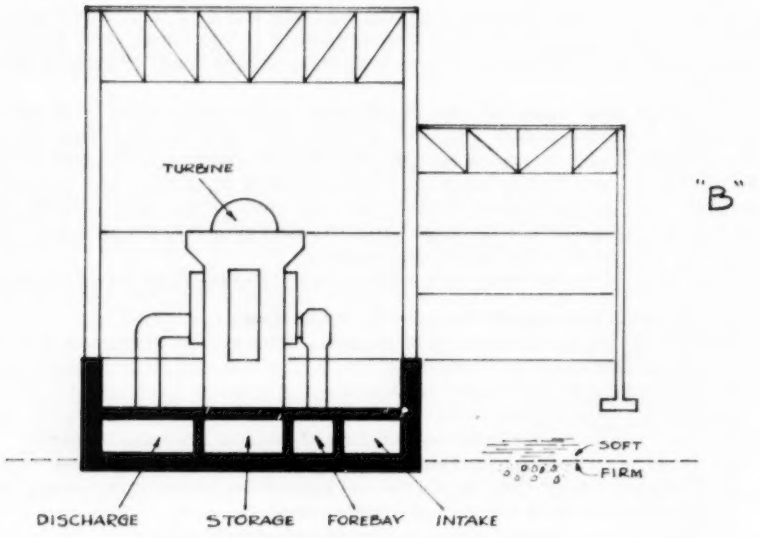
Also, if we adopt a light tone, we might say there are some "water heaters in the rear of the building." However, these are not ordinary water heaters; they are some 130 to 150 feet high, the equivalent of a 10 to 13 story building. What is more important to structural engineers is that these water heaters weigh some 10,000 tons and impose individual column loads of about 3,000,000 pounds.

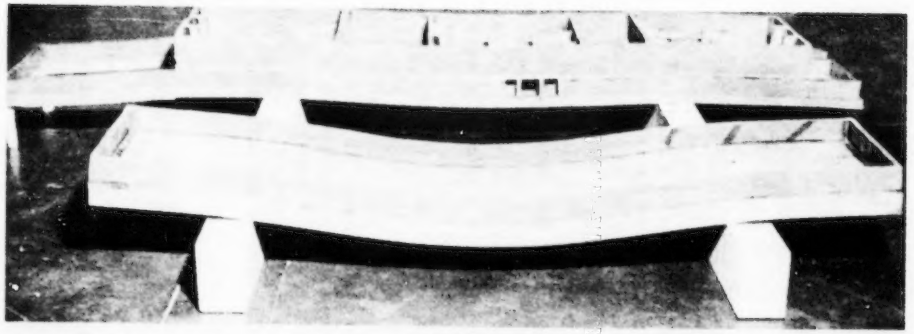
A list has been made of some of the many items which would be of interest, and a few features of each item will be amplified. It is believed that the selection of these topics is varied enough such that the readers, engineers in different activities, will find at least one of the topics of interest to him and can receive some information of value. This will require that each topic be discussed in general only, since each in itself could probably fill an entire article if we were to treat it thoroughly and in detail.

The basic topics of greatest Civil interest would probably be as follows:

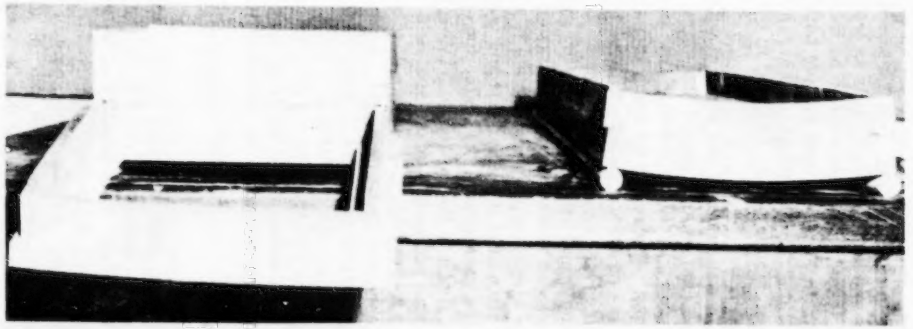
a) Foundation design of the hull-type foundation.
b) Dewatering and excavation of the site. This will be discussed in a paper on the Construction Problems.
c) Circulating water system intake tubes.
d) The Seismic factors which were considered.
e) The recent developments of reinforced grouted masonry utilized.
f) Turbine pedestal foundation design, empirical and dynamic considerations.
g) Stacks and their design, which went beyond the conventional static design in extensive wind vibration considerations.
h) Fuel oil tank retaining rings of prestressed concrete.


TURBINE
"A"
WATER STORAGE
INTAKE
DISCHARGE
FOREBAY
PILES (OR HEAVY MAT)


TURBINE
"B"
SOFT
FIRM
DISCHARGE
STORAGE
FOREBAY
INTAKE



SIDE VIEW



END VIEW

Rubber scale model of the Bathtub foundations at Contra Costa, showing comparison of conventional 8' thick mat type with the box or hull type that was designed and built to incorporate the necessary water passages.

Foundation Design

There were many varied foundation problems in the construction of the initial and the secondary extension to the Antioch or Contra Costa plant, and also in the separate plant under construction at Pittsburg. The most interesting one of the group is probably the so-called bath tub foundation for the turbine bay, the first stage of construction at the Contra Costa plant. It is as indicated on the Section Fig. 1-B. It was designed to carry the turbine foundation loads and building wall loads as a single foundation mat for lateral distribution of loads. The elements to be included in the foundation construction are as shown in Fig. 1-A, and include building foundation walls, intake tube, fore bays or distributing tubes, discharge tubes, turbine foundations and water storage area. These were combined in the final design as shown in revised section, Fig. No. 1-B.

Advantage was taken of the vertical walls and basement floor slab being designed and cast as an integral structure to distribute the lateral loads by Vierendeel or rigid frame action. This permitted the use of a relatively thin foundation mat. It was felt by some laymen engineers that the relatively thin slab might not be strong enough, although it could be proved by calculation that the Vierendeel section as designed was the equivalent of the solid 6' or 7' thick simple mat section. Therefore, to demonstrate this point more clearly, a simple rubber model was made, merely for qualitative demonstration that the structure as we proposed it was superior to a slab mat design. One model simulated the solid mat and one the continuous box frame. As can be seen in the photograph, the deflections of the mat type of foundation were many times those in the model simulating the combined built up girder structure which was proposed. Many analyses were made to verify that this was a satisfactory foundation, e. g. it was checked for hogging and sagging as a ship's hull would necessarily be designed. Again, these were qualitative rather than precise quantitative analyses to determine that the foundation we were providing met the factors of safety that were desired for the construction of this important public utility. The construction problems in the construction of this hull type foundation and the field problems such as dewatering will be discussed in a paper covering the construction difficulties of this program.

Intake Tubes

In order to provide adequate cooling water for the condensers in the turbine generator units, approximately 400,000 gallons of water per minute of cooling water are required. This quantity of water is required for cooling to maintain the high efficiency of the turbo-generator units by reducing the pressure on the discharge side. At this point some of the design features and description of the intake tubes will be made, but the discussion of the construction problems in installing and building this large system will be discussed in another article on the construction of this plant.

Tidal influence is experienced in the stream at the site and there would be a possibility that warm water discharged into the river, either upstream or downstream of the intake might flow toward the intake and be recirculated. In view of the importance of maintaining cool water at the condenser, the intake and discharge points were separated as noted on the plot plan of Contra Costa. They were placed far enough apart so that the backwash would not return to the intake until after it had been mixed with the stream.

Also, the principle employed was to place the intake tubes such that they collected cool water from near the bottom of the stream flow, and the discharge was made in a thin layer overlying the surface of the river to reduce the possibility of mixing the warm discharge water with the cool intake water required for efficient operation. This scheme at the Contra Costa plant has proved quite satisfactory. Investigation of the history of the stream contours at the Contra Costa site indicated that the stream bed adjacent to the site was relatively stable and that it would not be advisable to change the contours by dredging or other major operations. The designs of the inlet structures were, therefore, confined to such developments as would result in as little change in the stream bed as possible.

To reduce the height of the over-all intake structure, twin elements were placed side by side. They were originally planned to extend from the shore in a temporary dredged trench to a point intercepting the deeper portions of the stream channel with a flow line elevation of about 23 feet below mean low in low water. To achieve this depth at a point of contact with the normal stream bed required that the tubes extend to 400 feet out from shore into the river.

It was also necessary that a coarse bar rack structure be provided, with fine screening included because of the concern felt by the Fish and Game Commission that the small fish in this natural hatchery might be killed. This important, complicated and interesting problem of the fish segregation is a separate topic, discussed in a paper by Mr. J. A. Kerr.

One of the complicating features of the intake tube design is that provision must be made to permit dewatering of either or both of the tubes separately or simultaneously for inspection or any repairs that might be necessary. Due to these requirements for dewatering, the scheme selected was that of thick walled heavy concrete tubes 12 feet in inside diameter. Another factor influencing this design choice was the fact that there were forms available in Los Angeles already fabricated for some tubes of this size, and hence a considerable saving in form costs could be achieved. Pile supports were designed to carry the end of each 100 foot section and the gate structure as well as the transition section at the shore. The piles of course were considered for uplift forces as well as for vertical downward load capacity.

The Seismic Considerations

These introduced special considerations into the design problem primarily because of the magnitude of lateral force assumed. The gravity loads were heavy and the seismic coefficient, or lateral acceleration of gravity assumed, was relatively high.

Although the 1946 Uniform Building Code factor might have permitted 8% the design factor for these plants was assumed as 20%. It was assumed thus because it is much more important that a plant such as this be undamaged and continue to operate without interruption after a disaster than it would be for an ordinary industrial plant to continue uninterrupted operation.

To discuss the factor briefly, the 1933 earthquake at Long Beach and the recent Tehachapi earthquake may be mentioned. In neither case were the intensities of the earthquakes as great as that of the 1906 San Francisco earthquake. Yet, at Long Beach accelerations of about 0.3 g were measured.

A tank constructed near Tehachapi designed with a 10% factor nearly collapsed. The diagonal tie rods bracing the tank were stretched to such an extent that the paint had spalled and they stretched so that they were sagging

noticeably. An adjacent tank designed with a 20% factor suffered no distress nor damage.

This lateral factor was not applied to all the design live loads since these assumed live loads do not represent real loads that would be expected to exist in the plant at the time of a quake. The live load occupancy in a power plant is relatively low and may be neglected. This high factor was not applied to the piping loads in the plant because the piping loads in general are suspended from the structural frame by hangers, and do not impart lateral loads but tend to swing freely in case there is vibration or lateral motion of the ground.

Also piping loads from long runs of pipe cannot be assumed as being applied laterally at a constant factor, say 20%, to the anchors and supports. It would be quite possible for example, for the supports to move back and forth within the amplitude of the earthquake motion but yet not impart great lateral amplitude nor force to the central portion of the pipe length.

The boiler support was a unique problem. The tremendous weight of the drum and piping is suspended from the top of the frame some ten stories above the ground. It might be assumed, following our local code recommendations that 20% of the load carried at the top of the frame would be applied laterally at the top. This, of course, would indicate that there would be tremendous uplift forces and overturning moments to the frame. However, as all we who live with earthquakes know, the answer is quite different. For example, if we assume the weight all suspended at the top by hangers, we would in effect have a perfect seismograph imparting practically no lateral force at the top.

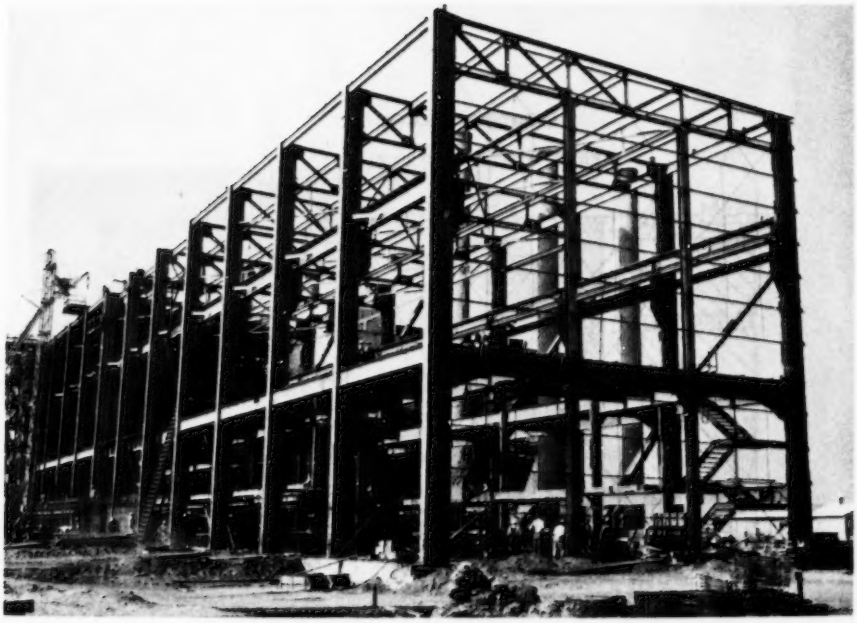
Actually, however, there are in the boiler frame and structure a series of stops which prevent side sway and a multiplicity of pipe connections which tie this otherwise free-swaying mass to the structure all the way up and down the frame. Therefore, the more rational assumption is that the lateral forces should be assumed as applied at the center of gravity of each mass wherever it occurs.

The basic scheme was relatively simple and conventional. It was simplified considerably by the fact that the loads were so heavy that X-bracing was not required. The bracing diagonals were so large to carry tension that they were large enough to carry compression also, having a good 1/r ration, i.e., a Warren-type bracing was used.

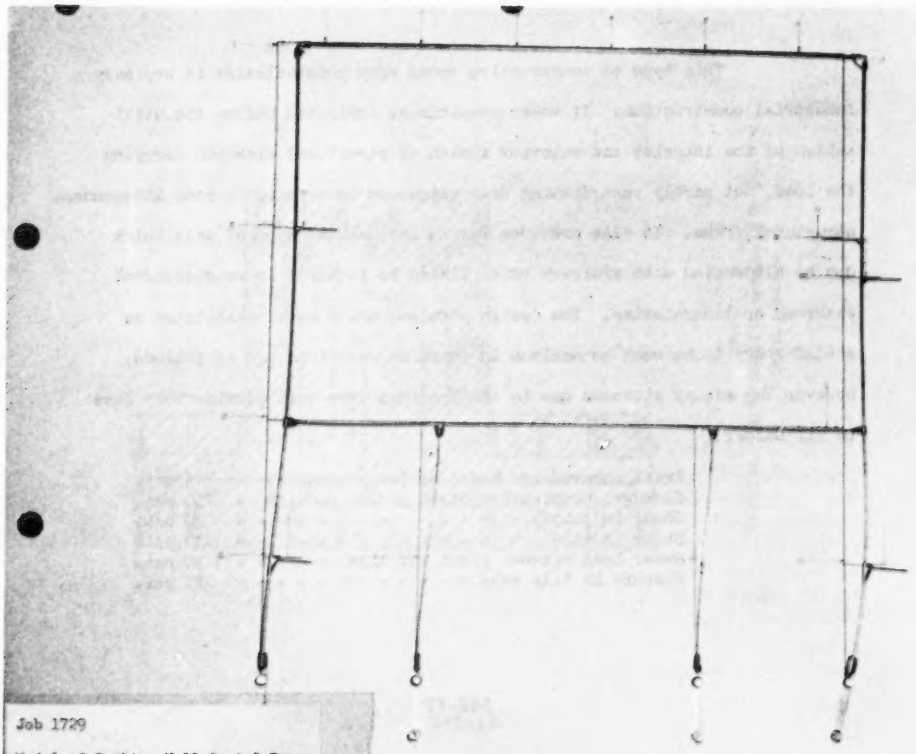
It was not possible to provide braces in the Turbine Hall extension of Contra Costa so the columns, roof trusses, and floor beams were designed as a complete rigid frame, continuous in all joints and fixed at the base by the heavy foundation construction. This is as shown on the erection progress photograph. The simplified frame is shown more clearly in the photo of the wire model of the frame bent.

The model was used to check the necessary indeterminate frame calculations for the many different loading conditions. It was made of brass wire calculated so the product of Modulus of Elasticity times the Moment of Inertia of the wire was proportional to that of the frame.

The principle used was the same as is used in the conventional Beggs Deformeter or Gottschaulk Continostat, i.e., "let the frame draw its own influence line of reactions." The technique has been simplified however so that little time and little cost of equipment is involved. The material for this particular one cost $.97. About a day was required to make the frame and draw the influence lines of all vertical, horizontal and moment reactions. It was judged that the accuracy was as well within 10% as were the comparable mathematical calculations.



CONSOLIDATED EDISON STEAM ELECTRIC PLANT - E.S. & D. JOB 1729 - Steel frame for turbine hall being erected. Moment connections are shown at operating floor level beam.

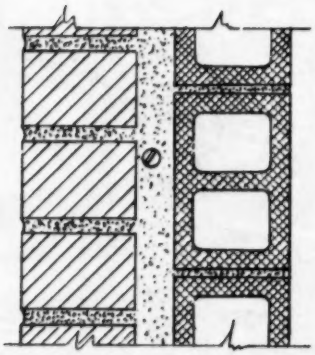


Job 1729

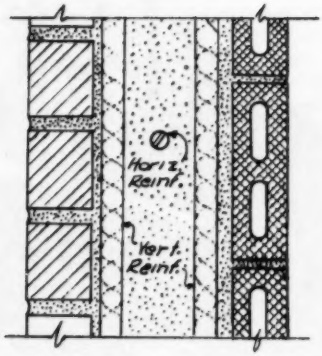
Model of Turbine Hall Portal Frame shown distorted to trace influence line for moment at right reaction.

REINFORCED GROUTED

BRICK & TILE CONSTRUCTION



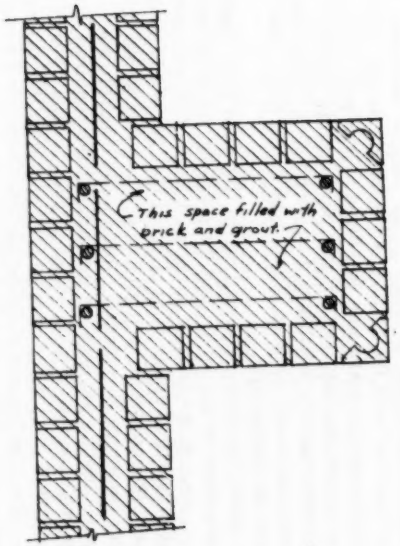
TYPICAL VERTICAL WALL SECTION



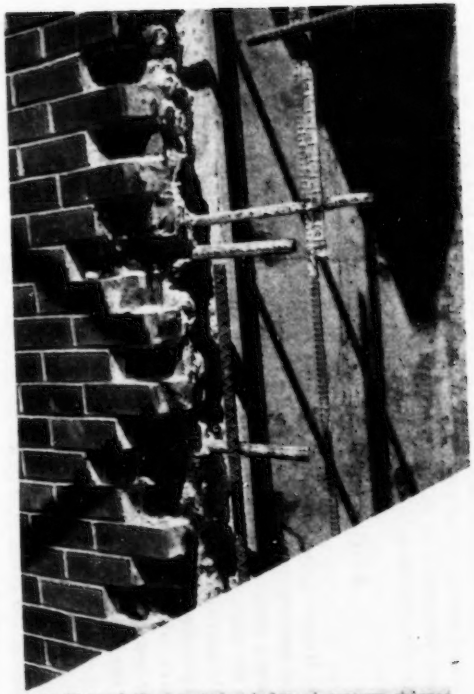
VERTICAL SECTION AT PILASTER

This type of construction opens many possibilities in ordinary industrial construction. It makes possible as indicated before the utilization of the interior and exterior finish as structural elements carrying the load, not merely contributing dead weight to be carried by some independent structural frame. It also provides for an inexpensive type of wall which can be alternated with ordinary brick finish as required in an occasional washroom or laboratories. The design stresses which were established as satisfactory to be used as maximum in these calculations are as follows, however the actual stresses due to the loadings were very considerably less in all cases:

Axial compression based on gross section - -	125 psi.
Flexural compression based on net section -	250 psi.
Shear in joints - - - - - - - - - - - - - -	20 psi.
Shear in tile - - - - - - - - - - - - - - -	20 psi.
Shear bond between grout and tile - - - - -	30 psi.
Flexure in tile webs - - - - - - - - - - - -	75 psi.



VERTICAL SECTION
thru
TYPICAL REINFORCED GROUTED BRICK
BOND BEAM



A portion of the grouted reinforced masonry chipped away to receive the new extension of the wall. Note the bond between the brick ties and the concrete grout core.

Effective though it is, the procedure is simple.

(a) Wires are calibrated by measuring deflection due to a standard pull. This gives a measure of combined EI.
(b) Solder the properly proportioned wires to model, to a scale so deflections are easily measured.
(c) Deflect the model by displacement of the points of reactions.
(d) Trace the resulting influence lines.

Reinforced Masonry Construction

The basic frame of the building at Contra Costa was structural steel, providing lateral and vertical support for the concrete floors and lateral support for the masonry walls. The frame of the turbine bay and adjacent bay is about as shown in the photographs of the wire model which were made to check the indeterminate calculations necessary to arrive at the design of this frame. The walls span between these frames for a distance of 27' from column to column to resist heavy lateral earthquake loads. There were no steel beams and only one concrete beam incorporated in the masonry; the bond beam function being provided by the reinforcing placed in the masonry as part of the bricklayers' work.

Again the size of this project constituted one of the more unique features. This is one of the largest reinforced masonry jobs in this area - something over 1,000,000 of the large size brick being used. The 27' horizontal span of the walls is somewhat larger than normal and the 80' height of the walls is considerably above the average. The scheme of masonry was that of grouted reinforced brickwork using the Port Costa Keybrick available locally. These are essentially common brick in one face of which is a dove-tailed key. The outer and inner tier of brick are laid and the reinforcing is placed, then the grout is poured in the space between the two tiers as the work progresses. Bond between this inner collar of reinforced grout, or concrete, and the masonry surfaces is provided not only by adhesion of the grout to the porous brick but also by the mechanical key provided on the back of the brick. As many of you who are familiar with the history of structural design in California know, this type of reinforced masonry was developed in Southern California, the scene of several serious earthquakes. It was developed primarily to provide a type of reinforced masonry which would be sound structurally and at the same time economical to lay. Details of this type construction are shown in the sketches and in the photograph.

The reasons for selection of this type wall were: the good cost comparison with concrete, the low maintenance costs, and the pleasing, dignified and interesting appearance. One of the greater advantages is the fact that there is a continuous, unbroken, vertical collar joint of grout that serves as an effective moisture barrier. There are no through headers nor ties which would be potential passages for moisture penetration. Reinforced concrete bond beams were not used generally on this job because that would have required that the masonry crews stop, wait for the forming crews to place forms, then that concrete crews place concrete, then that forms be stripped, then that masonry work proceed again. The starting and stopping and coordination of these trades causes unpredictable delays which increase the intangible costs of construction. The typical bond beam construction was of masonry in sections somewhat as shown in the photograph. The strength of such walls, properly built compares well with concrete structures.

This might seem improbable to the average engineer accustomed to concrete technology where the high water-cement ratios so typical of proper masonry grout would certainly result in low cylinder tests. However, three important factors are effective here. First, it should be pointed out that the grout is really 6 or 7 sack concrete and the water cement ratio is approximately 6 to 7 gallons per sack. The second and by far a more important factor to be noted is that the grout is poured between two absorbent tiers of brick which promptly withdraw a portion of the water from the grout, reducing the water-cement ratio to the no-slump value of 3 to 4 gallons per sack, which compares favorably with vacuum processed concrete. The third factor is that, although a portion of this water migrates to the exterior of the wall and evaporates, the balance is retained and this, in effect, acts to "wet cure" the grout. This grout which tests up to only 1500 or 2000 psi when conventional wet cylinders are made will test anywhere from 3000 to 6000 psi when tested under simulated job conditions.

Tests demonstrated qualitatively that the theory was correct. The grout would increase in strength as a result of lowered water-cement ratio and improved curing.

The effectiveness of this reinforced grouted masonry was further utilized in combination brick and glazed tile walls for various parts of the Pittsburg Steam Plant. The interior walls in general were of brick but in cases of washrooms and laboratories this interior finish was of glazed structural clay tile. This made a composite brick-grout-tile wall in which the exterior finish and the interior finish served not merely as dead weight contributing to load on the building, but served as a portion of the structural element helping to carry load and could be constructed by the same trade without costly interruption or conflict. The section through the composite wall is as shown in the drawing. This required that rather complicated rigid frame analyses be considered on the cross-section of the wall. To further check the calculations, however, beams of the structural clay tile and the brick work were built and tested, resulting in values considerably higher than those calculated.

The Turbine Pedestal Foundation Design

The structural design of power plants involves considerable research and study, but no part of the station is more important than the support for the turbines and generators, the "heart" of the plant. Although good engineering practice generally requires that structures be economically designed, greater factors of safety and more conservative consideration must be given to the turbo-generator supports, commonly called Turbine Pedestals. The support of these turbines is not only a static problem of supporting some 700 tons of equipment but also of the deflections and the probability of resonant vibrations which might occur in the structure. The stress considerations of the static loads in general are not very serious. The probability of vibrations is one of the reasons it is generally desirable that turbine pedestals be made of concrete rather than of steel. The concrete has a better vibration damping factor than does the more resilient steel. These structures are generally rather large and complicated shapes, such that accurate calculations are very difficult. In general, most of the rules governing the design are empirical, and are based on what the leading turbine generator manufacturers have found to be good practice in the design of foundations. Therefore, designs are generally laid out having in mind the use of schemes which are similar to previously built foundations which have proven satisfactory. Some of the general rules

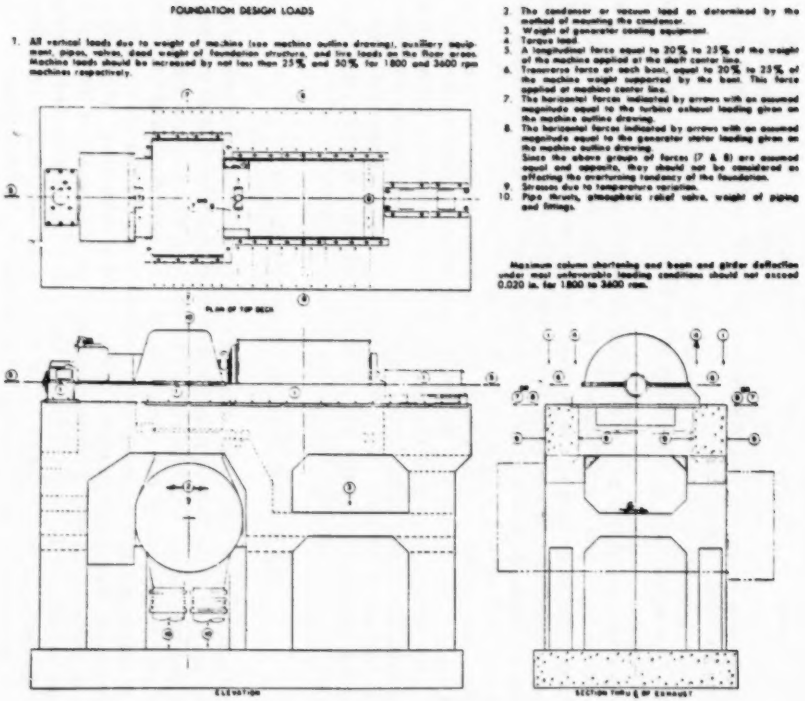

FOUNDATION DESIGN LOADS
1. All vertical loads due to weight of machine (see machine outline drawing), auxiliary equipment, piping, valves, dead weight of foundation structure, and live loads on the floor areas. Machine loads should be increased by not less than 25% and 50% for 1800 and 3600 rpm machines respectively.
2. The condenser or vacuum load as determined by the method of mounting the condenser.
3. Weight of generator cooling equipment.
4. Torque load.
5. A longitudinal force equal to 20% to 25% of the weight of the machine applied at the shaft center line.
6. Transverse force at each bent, equal to 20% to 25% of the machine weight supported by the bent. This force applied at machine center line.
7. The horizontal forces indicated by arrows with an assumed magnitude equal to the turbine exhaust loading given on the machine outline drawing.
8. The horizontal forces indicated by arrows with an assumed magnitude equal to the generator stator loading given on the machine outline drawing.
Since the above groups of forces (7 & 8) are assumed equal and opposite, they should not be considered as affecting the overturning tendency of the foundation.
9. Stresses due to temperature variation.
10. Pipe thrusts, atmospheric relief valve, weight of piping and fittings.
Maximum column shortening and beam and girder deflection under most unfavorable loading conditions should not exceed 0.020 in. for 1800 to 3600 rpm.
PLAN OF TOP DECK
ELEVATION
SECTION THRU EXHAUST

Fig. 1. Foundation loading diagram

to follow are that thin slabs or walls and cantilever brackets are to be avoided; generous haunches are provided; large masses of concrete are provided with respect to the size of the equipment to be handled; for example, experience has indicated that concrete mass at least equal to 2-1/2 times the weight of rotating parts should be provided. It is customary to limit the elastic deflections of the piers and girders to a maxim of 0.020 inches and to further check that the relation between mechanical frequency and deflection should be such that there is a minimum of probability of the machine and the turbine pedestal being in tune. This has been expressed by the General Electric Company as an equation stating: the frequency in cycles permitted equals 187.7 divided by the square root of the deflection in inches. Therefore, for the following speeds or rpm of various motors the following theoretical deflections for resonant vibrations should be avoided.

Speed	Theoretical Deflection for Resonant Vibration
1200 rpm	0.025 in.
1500 rpm	0.015 in.
1800 rpm	0.011 in.
3000 rpm	0.004 in.
3600 rpm	0.0028 in.

Although most of the vibration information available on turbine pedestal foundations is the rule of thumb type of equation, there are many publications such as the articles by Rausch, Kayser, Ehlers and others which appeared in the German technical press. These rule of thumb guides may be tempered by study of this foreign literature and Mr. Thon, Assistant Civil Engineer, Power Division, has translated several of the authentic German studies which were made on the correct basis of analysis in order to further assure us that these pedestals will be soundly designed.

Evidence of the soundness of the design was indicated by a comment made by one of the construction men on the job after the plant had started operation. He pointed to the rubber separation which exists between the turbine pedestal and the rest of the structure and said, "I thought at one time that you put this rubber strip in to prevent turbine vibrations from going into the building. However, it seems that this rubber strip prevents the building vibrations from going into the turbine pedestal. There was less vibration noticed on the turbine pedestal. There was less vibration noticed on the turbine pedestal than there was on the rest of the floor."

The foundation loads of this particular piece of equipment, namely the turbine and generator for which the entire plant is to be constructed are as follows, and as further shown in the loading diagram:

Vertical - The total dead load of about 700 tons to which is added 35% which is called impact is actually to provide an additional factor of safety in the event of dynamic imbalance.

Steam Condenser Vacuum Load

Torque Load - This is due to changes in load, acceleration, etc. and amounts to about 7000 x kw ÷ the rps, and for a machine of this capacity amounts to about 1,000,000 pounds which is used for design purposes.

External Horizontal Load - It is assumed that horizontal loads, crosswise and lengthwise of the longitudinal axis of the machine, are loads supplied at the center of gravity of the shaft, and these loads amount to 20-25% of the weight of the machine. Guides are provided to resist such forces and to

maintain the various portions of the turbine generator unit in proper alliance. Again we find that maintenance of alignment or minimum deflection, is more important than consideration of stress.

Earthquake loads due to the condenser weight - These must be trasmitted to the main body of the machine through the "neck" of the condenser. This, however, is not generally of great concern because of the low stresses involved.

Temperature Stresses - These are indeterminable and therefore reinforcing for this purpose is based on practice followed in structures which have proven satisfactory in this respect.

The stresses to resist all the above loads are kept low since deflections are of great concern, especially as they influence vibrations.

The Chimney or Stack Vibration Problem

The boiler chimneys or stacks presented an interesting design and erection problem. The erection features and fabrication will be described in another paper on the construction. These were straight cylindrical stacks, 11' in diameter for the first six boilers at Contra Costa; 16'6" in diameter for the second two stacks; and 18' in diameter for the stacks at the Pittsburg site. All are 200' high, lined with insulating gunite to provide protection against possible corrosion due to flue gas, and to provide insulation for proper chimney action. The design problems were interesting and complicated. At the time the particular stacks at Contra Costa were being designed, another set of stacks somewhat similar at another plant were subjected to considerable distress due to resonant vibration in wind, and the problem was then to establish that the stacks as designed for Contra Costa were satisfactory for such conditions.

There have been frequent examples of such stack distress. However we find very little mention of these in publications. Apparently we engineers are not prone to brag about our failures. A rather extensive search of literature on the subject indicated that, in general, stacks have been built successfully by various tricks and devices, but there was very little of research on stacks suffering distress, which may be of the type shown in photo.

Wind, particularly where subject to uniform conditions so that steady laminar flow may exist, may peel off a cylindrical body alternately from one side then the other, imparting a pulsating force at right angles to the wind direction. The period of this pulsating, alternating force depends upon the velocity of the wind and the diameter of the cylinder and is expressed by the equation:

$$f = \frac{SV}{d}$$

f = frequency, - cycles/sec
V = Velocity, - ft. per sec
d = diameter, - feet
S = Strouhal number, approximately 0.20

This is a phenomenon known as Von Karman vortexes. If the frequency of these forces occurs at the natural period of the stack, the energy from each impulse will, of course, accumulate, just as a slight push on a child's swing at the proper time will accumulate energy until both would vibrate or swing to relatively large amplitudes. Swinging will continue to build up in amplitude until damping is sufficient to dissipate the energy of each input. The damping is due to air friction, internal friction in the steel, internal friction in the

lining if it exists, and dissipation of energy due to base rotation, or due to friction on various attachments such as breeching connections.

There was little basic information on the subject available. The equations were not well established for such phenomena. There was some test data available which established the damping coefficient due to internal friction in steel, and in some materials such as aluminum and bakelite, but no authentic value in common publications on gunite and concrete. It was necessary to develop values for the exponential constants in the equation.

A model stack was constructed and tests were run to determine the vibration damping characteristics of steel and gunite. These tests gave decadence curves as shown on the chart. Calculations were made using the equations which were developed and the constants were checked by running full scale vibration tests on the stacks which had experienced the initial vibration distress. Also, calculations were made on the Contra Costa stacks and further vibration tests were made on those stacks, lined and unlined.

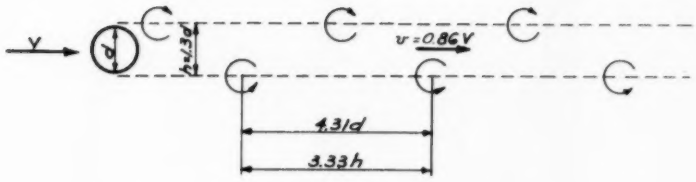
The final tests on the stacks at Contra Costa consisted of rigging between the two stacks at the top and pulling them over toward each other as one might twang a tuning fork. The stacks were pulled approximately a foot out of line and released with a quick release trip mechanism. Deflection readings were taken by recording seismograph installed at the top of the stack, by a pantograph recording needle on a scaffold about 60' above the base, by electric strain gauges at the base, and Ames dials mounted on the base to determine base rotation. These furnished attentuation curves of the decadence as shown on the chart enclosed. Although the principle of damping due to dissipation of energy by internal friction of the material of the lining was employed at Contra Costa, there could have been other methods used such as:

a) Guy wires. These would not be very effective without being excessively large. One approximate check indicated 3" diameter cables would be required.
b) Vibration Dampeners. These have been used successfully to damp generator vibrations and washing machines, - but would have such large masses and amplitudes themselves that they would not be feasible on these stacks.
c) Rigid trusses. would be uneconomical and difficult in view of the flexibility that must be maintained for differential expansion, one stack to the other, as imposed by the temperature changes.
d) Trussing - with friction damping connections could be used. This method was expensive and would be subject to possible freezing of the moving parts and, hence, ineffectiveness of the principle.
e) Miscellaneous. Methods might be devised to prevent the rythmic formation of the vortexes. This is now the subject of a graduate research program at Stanford University.

This principle required the use of the design and test data which was mentioned earlier in this article. The outline of design procedure in this and subsequent stacks was more complicated than normally would be employed for simple stacks. The procedure was basically.

1) Determine the period of the stacks designed by conventional assumptions.
2) Determine the corresponding resonant wind velocity.
3) Determine the forces or energy imparted to these stacks by the wind.
4) Determine the amplitude at which the dissipation of energy by base rotation and internal stress friction would be as great as the forces put in, so there will be no increase in amplitude.

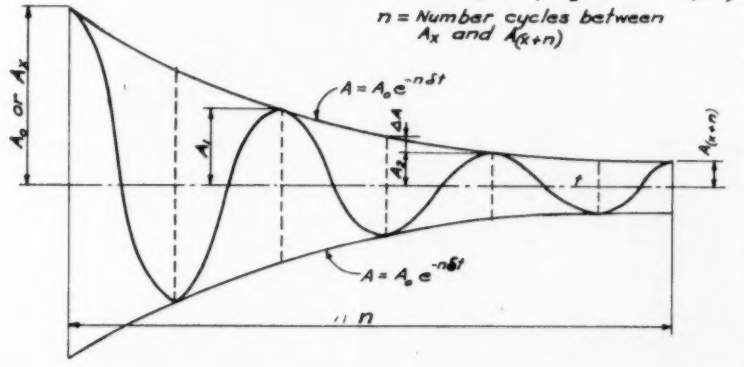
PHOTOGRAPH OF 12 FT. DIAMETER STEEL STACK. THIS WAS SUBJECTED TO WIND VELOCITY APPROXIMATELY RESONANT WITH THE PERIOD OF THE STACK WHILE UNLINED. THE FAILURE WAS BY BUCKLING FOLLOWED BY TEARING OF THE PLATE.



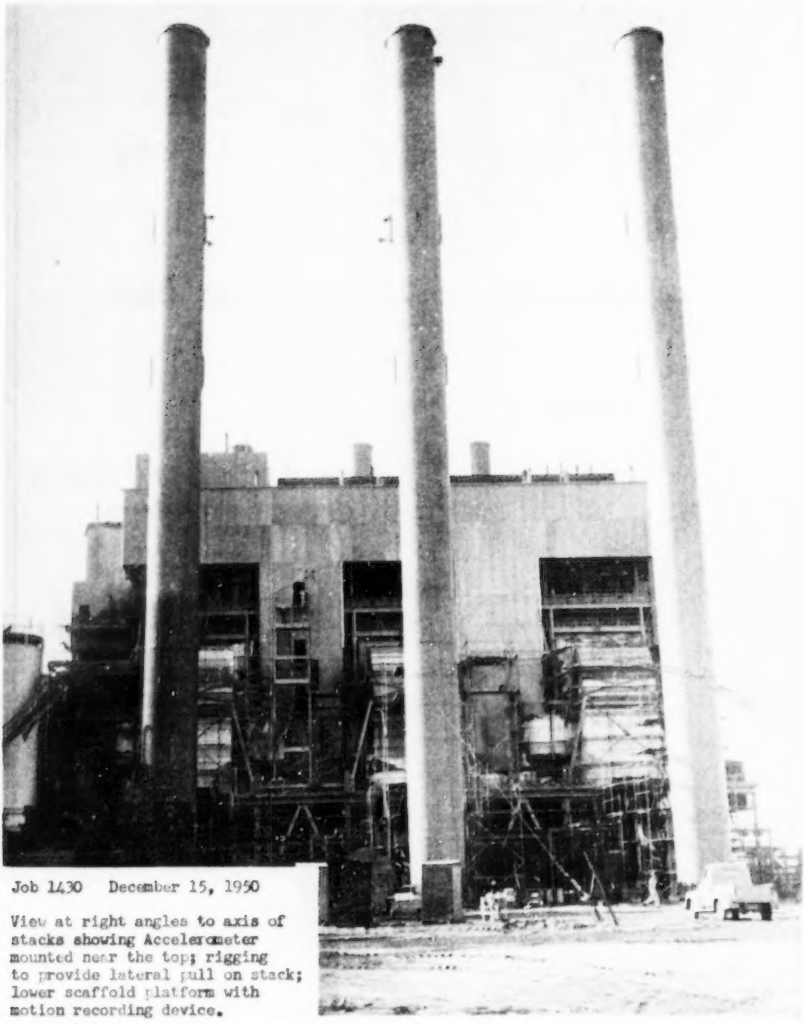
KÁRMÁN VORTEX TRAIL IN WAKE OF CYLINDER

$$\delta = \text{Log}_e \frac{A_2}{A_1} \approx \frac{2\,\Delta A}{A_1 + A_2} = \frac{1}{n}\left(\text{Log}_e \frac{A_x}{A_{(x+n)}}\right)$$

n = Number cycles between A_x and $A_{(x+n)}$

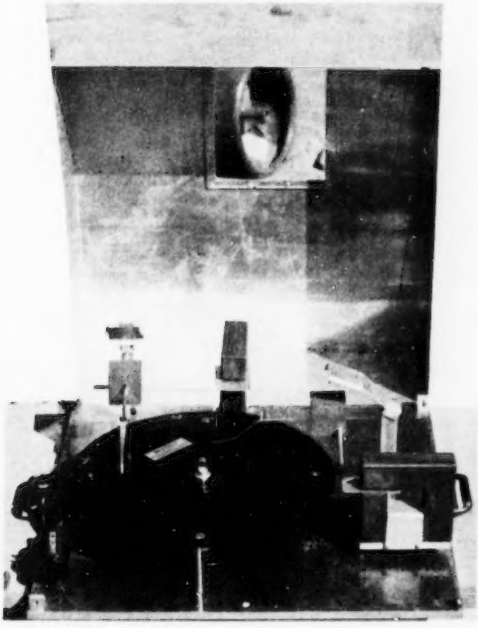


DECAY CURVE

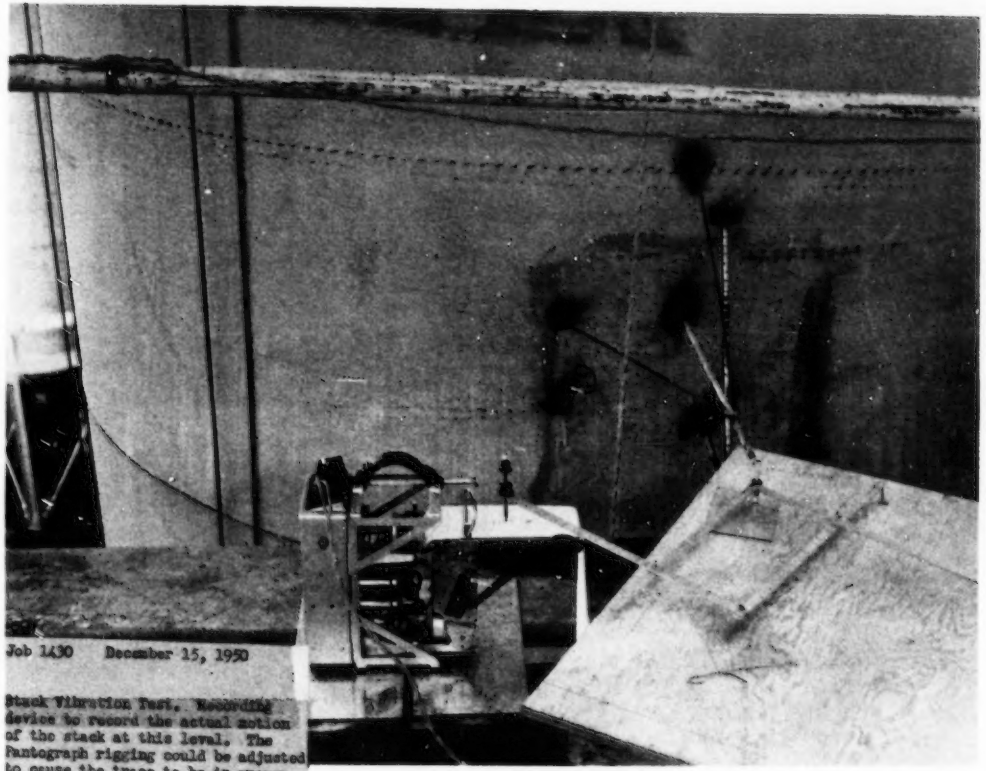


Job 1430 December 15, 1950

View at right angles to axis of stacks showing Accelerometer mounted near the top; rigging to provide lateral pull on stack; lower scaffold platform with motion recording device.



Seismograph which was mounted at the top of the stack to record the vibrations. It is shown with cover raised to show the arms to record on the smoked disc the components at right angles to each other.



Job 1430 December 15, 1950

Stack Vibration Test. Recording device to record the actual motion of the stack at this level. The Pantograph rigging could be adjusted to cause the trace to be in proper scale for the chart.

This requires rather complicated calculations and for a simpler problem such as when stacks are obviously quite conservatively designed or when stacks are shorter or smaller, this method of calculation has been simplified as will be indicated in a paper soon to be published on this same topic.

Basically the principle, in the simplified calculation, is to determine the period of the stack and the resonant wind velocity, detail the stack in such a way that the resonant vibration would not occur with a wind velocity of about 40 miles per hour. A simple assumption of a lateral force per impulse is assumed and then for unlined stacks a magnification factor of 15 or 20 is assumed and for lined stacks a magnification factor of about 10 is assumed.

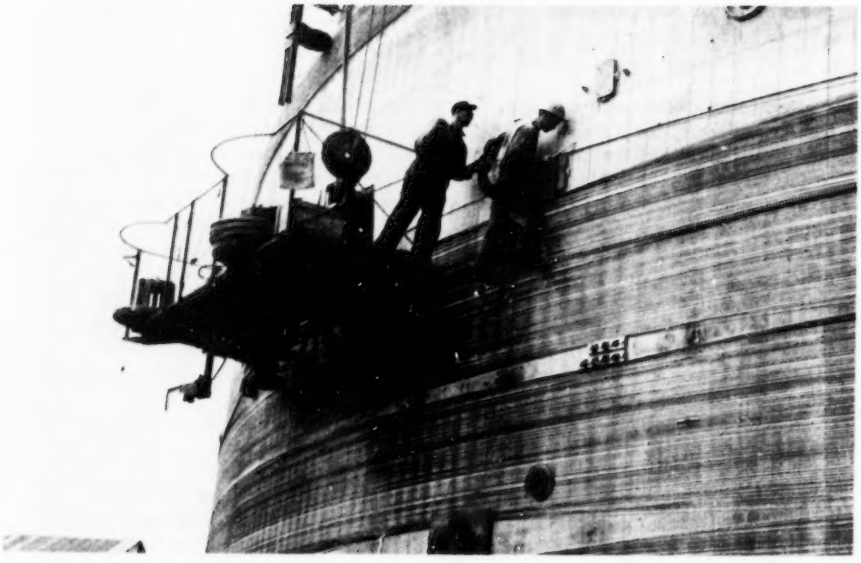
Fuel Oil Retaining Walls

Retaining walls were required around the fuel oil tanks to provide the customary capacity for retention of the oil in the event of tank failure or of fire. Due to the fact that the conditions at the Pittsburg site were congested and due to the fact that there was not earth fill material locally economically available, it was found by a comparison of various types of retaining walls that the most economical, though somewhat unusual, solution would be circular rings of prestressed concrete. These were of the Pre-load method as placed by the Pacific Bridge Company of San Francisco, such as has frequently been used on water treatment tanks or other large concrete tanks such as sewage digesters. Although leakage was not a serious consideration in these large retention walls, the consideration of economy indicated that prestressing by a high-strength wire would be the most desirable type of reinforcing, and at the same time achieve a wall free from shrinkage cracks.

A further unusual development resulted from the economic investigation. Rather than pour these circular rings as sectional segments poured in place or as a slip-form type of construction, the rings could be built up as a series of pre-cast flat segments forming a multi-sided polygon. Due to the large diameter of these rings, namely 300'-0", it was found that flat segments of 23'-8" high by 9'-3" wide could be satisfactorily erected as "staves" and then held in place by the wire wrapping. The resulting section of the wall is as shown in the sketch which includes the gunite protection placed over the high strength wire in order to provide protection. The joints between these vertical "staves" were grout-filled joints to provide good bearing. The thickness of the wall shown was necessary in order to resist the compression resulting from tension in the pre-stressing wires.

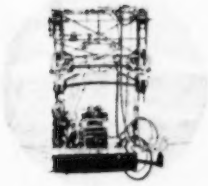
The design specified high strength wires of approximately 200,000 p.s.i. to be placed with an initial tension of about 140,000 p.s.i. This initial tension would permit ample margin for decrease of stress due to shrinkage, plastic flow and yield. The most economical method of placing was by the Pre-load method. In this method a "carriage" travels around the tank stressing the .162" diameter wire by pulling it through a .144" diameter opening.

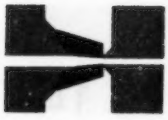
Additional data on this method is shown on the accompanying illustrations.



PHOTOGRAPH OF WIRE STRESSING CARRIAGE, WHICH DERIVES MOTIVE TRACTION FROM A CHAIN DRIVE TO WRAP THE WIRE FROM THE REELS ONTO THE TANK. THE SPREAD OF WIRES TO MISS SLEEVES CAN BE SEEN, ALSO A SPLICE IN THE WIRE AT THE LOWER RIGHT.

Special features of the prestress system



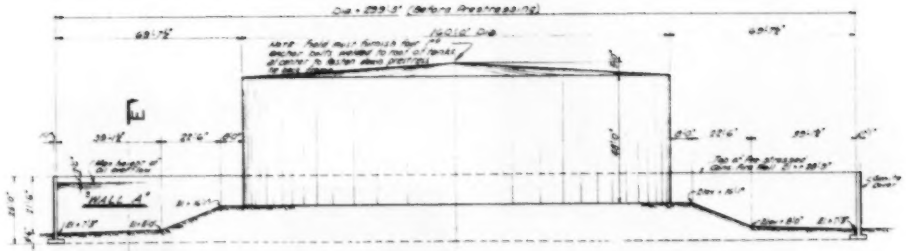


1. High tensile strength steel wire, similar to suspension bridge cable wire. It is capable of withstanding a pull of 175,000 pounds per square inch before permanently stretching and of more than 200,000 pounds per square inch before breaking.

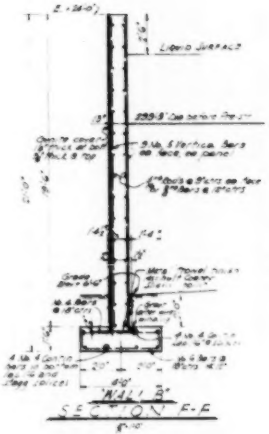
2. Pulling the special wire (.162") through a die of smaller diameter (.144") at the time of its installation. This tensions the wire uniformly to 140,000 pounds per square inch.

3. A machine, supported by cables from a revolving rig on top of the tank, which travels around the outside of the tank shell to place the tensioned wire on the tank wall at uniform spacing.

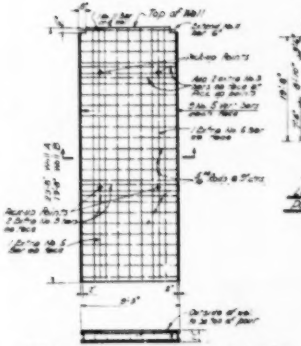
4. Covering the outside wire wrap and the inside steel rods with gunite—an air-applied dense concrete mortar—to protect the steel and give the tank a smooth finish. This cover coat bonds tenaciously to both the steel and the concrete.



FUEL OIL STORAGE TANKS — TANKS No 1, 2, 3 & 4
TYPICAL ELEVATION

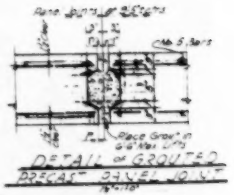


SECTION E-E

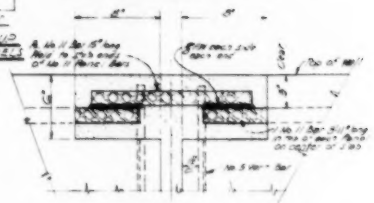


TYPICAL PRECAST WALL PANEL

LOCATION OF PICK-UP POINTS FOR WALL PANELS



DETAIL OF GROUTED PRECAST PANEL JOINT



DETAIL OF PANEL TIES AT TOP OF WALL